APPENDIX I

FINAL PRELIMINARY HYDRAULICS MODELING REPORT

URS

April 6, 2005

South Florida Water Management District
3301 Gun Club Road
West Palm Beach, FL 33416-4680

Attn: Ms. Maria Clemente, PE

Re: WO 01 Revision 1 Submittal
Final Preliminary Hydraulic Modeling Report
Stormwater Treatment Areas 5/6 Expansions Design
Contract CN040936

Dear Ms. Clemente,

Please find enclosed, the Final Preliminary Report results of the Hydraulic Modeling and Water Quality performance assessments performed per Amendment 1 of Work Order 01. This Final Report also includes the results of DMSTA Modeling performed for the nutrient uptake performance assessment of the cells and incorporates responses to the second round of comments provided by the District.

The following report also includes a number of recommendations relative to structure sizing, removal/replacement of existing structures and bridges and presents preliminary cost estimates for the construction and operation of STA-5 Flow-way 3, STA-6 Section 2 and the modifications to STA-6 Section 1 Cells 3/5. Additionally, preliminary cost estimates are presented for construction of the C-139 Annex 452 CFS pump station and gravity discharge culverts from the reservoir.

Attached in Appendix G are the responses to several reviewers' comments to the second submittal of comments on our report provided to the District on March 10, 2005. Responses to those comments as appropriate, have been incorporated into the Final reported transmitted herein.

Respectfully Submitted,

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Vice President

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Senior Project Engineer/Modeler

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TABLE OF CONTENTS

7800 Congress Ave.
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Tables

7800 Congress Ave.
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Figures

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Appendices

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List of Acronyms

BMP	Best Management Practices
CADD	Computer Aided Design Drafting
CERP	Comprehensive Everglades Restoration Program
CFS	Cubic Feet per Second
cm	Centimeters
DBHYDRO	Districts Data Base Hydro
EAA	Everglades Agricultural Area
EFA	Everglades Forever Act
EPA	Everglades Protection Area
FESWMS	Finite Element Surface Water Modeling System
FDEP	Florida Department of Environmental Protection
HEC-HMS	Hydrologic Engineering Center – Hydrologic Modeling System
HEC-RAS	Hydrologic Engineering Center- River Analysis System
hm3/day	Million cubic meters per day
Kw/Hr	Kilowatts per Hour
LTP	Long Term Plan
NAVD	North American Vertical Datum
NGVD	National Geodetic Vertical Datum
ppb	Parts per billion
ppm	Parts per million
SAV	Sub-Aquatic Vegetation
SMS	Surface-water Modeling System
SPF	Standard Project Flood
SPS	Standard Project Storm
STA	Stormwater Treatment Area
USACE	United States Army Corp of Engineers
USSC	United States Sugar Corporation
WSE	Water Surface Elevation

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PRELIMINARY HYDRAULICS MODELING REPORT
Stormwater Treatment Area (STA) 5/6 Expansions Design
Contract CN040936, Work Order WO-01, Task 4.2

EXECUTIVE SUMMARY

The build out of the STA 5/6 system will provide significant improvement in the ability to treat stormwater flows from the C-139 and C-139 Annex Basins in Hendry County, Florida. The current STA-5 Flow ways 1 and 2 and STA-6 Section 1 systems will be expanded into the Compartment C parcel (USSC Unit 2) located south of STA-5 Flow way 2 and north of STA-6 Section 1 including the previously planned construction of STA-6 Section 2.

As part of the development process, preliminary hydraulic modeling and engineering of the proposed STA expansions have been conducted as part of the design process. The following project features and findings are identified.

HYDRAULIC MODELING

- The routing and distribution of flows in the vicinity of the STA-5/6 area have been assessed and the Design and Standard Project Flood flows have been quantified through calibrated modeling.
- Hydraulic modeling of the canal systems surrounding the STA-5/6 area and Water Conservation Area 3 (WCA-3A) has been conducted to assess the effect of implementing the expanded STA-5 and STA-6 systems on the existing flood protection levels within the C-139 Basin.
- Hydraulic modeling of all the proposed STA cells has been conducted to assess the flow patterns and head losses within the cells.
- The flow from the C-139 Annex Reservoir to the L-3 Canal can be transmitted by gravity flow through gated, twin-barrel 8' by 6' box culvert under some basin conditions. Under some flood conditions, a 452 cfs pump will be required to discharge the C-139 Annex flows to the L-3 Canal for inflow and treatment primarily to the STA-6 system.

STRUCTURE ADDITIONS AND MODIFICATIONS

Initial Configuration

- The Initial Configuration of the treatment system is anticipated to require 16 new hydraulic control gravity structures to manipulate flows through the proposed new treatment flow-ways. These structures are anticipated to be electrically operated, remotely controlled, gated concrete box culverts with throat sizes ranging from 8' by 6' to 10' by 10'.
- A gated, twin-barrel 10' by 9' box culvert diversion structure (G-407A) will be needed on the L-3 Canal at the south end of the project area to limit flows to WCA-3A.
- The existing G-406 diversion structure located on the L-3 Canal near the north end of the project area will need to be modified by lowering the entire crest of the earthen ditch plug by about 1.25'.
- About 2.5 miles of drainage canals will be constructed along the eastern boundary of the project area to serve as a flow outlet for the treatment areas. These canals will be north-south extensions of the existing STA-5 and STA-6 Discharge Canals. The southern portion of the existing STA-6 Discharge Canal will require expansion to handle the increased proposed flows under the buildout condition.
- The G-607 culvert structure located at the confluence of the STA-6 Discharge Canal and the L-4 Canal will require removal and replacement with either a high capacity culvert or bridge to accommodate the proposed discharge canal flows.
- The G-88 culvert structure located at the western end of the L-4 Canal at its intersection with the L-3 Canal will require demolition and removal to accommodate the proposed flows to WCA-3A. As an alternative to demolition of these structures, a channel could be excavated through the L-4 levee to directly connect the STA-6 Discharge Canal to the L-3 Extension Canal. A new bridge will be required at this location if continued access to the levee/island area between the L-4 Canal and L-3 Extension Canal is required.
- The G-155 weir structure located at the western end of the L-3 Extension Canal will require demolition and removal to accommodate proposed flows to WCA-3.

<u>Build Out Configuration</u>

- The Build Out Configuration of the treatment system is anticipated to require 15 new hydraulic control gravity structures to manipulate flows through the proposed new treatment flow ways. These structures are anticipated to be electrically operated, remotely controlled, gated concrete box culverts with throat sizes ranging from 8' by 6' to 10' by 10'.
- About 2.5 miles of additional drainage canals will be constructed along the eastern boundary of the project area to serve as a flow outlet for the treatment areas. These canals will be north-south extensions of the existing STA-5 and STA-6 Discharge Canals.
- A gated box culvert structure (G-407B) will be constructed on the Discharge Canals to provide a controllable connection between the STA-5 and STA-6 segments for flexibility in routing treated effluent to the north or south of the Build Out treatment area. The size of the structure will be assessed during the Compartment C buildout design phase.

- A large discharge pumping station will be required on the STA-6 Discharge Canal to handle the proposed flows when the system is expanded to its Build Out configuration. The pump station peak capacity is estimated to be about 2,812 cfs.
- The existing G-600 pump station will be demolished during the Build Out phase of the treatment system construction.

WATER QUALITY IMPROVEMENTS

- Water quality modeling of the expanded treatment system has been conducted using DMSTA software to assess the level of phosphorus removal from the influent waters and the resulting effluent water quality. Based on a simulation period of at least thirty years, the completed system is estimated to remove an average annual load of 35,000 to 35,700 kg of phosphorus and the outflow phosphorus concentration is estimated to range between 11 and 15 ppb (geometric mean).
- DMSTA modeling indicates that there is minimal benefit to constructing interior levees in either or both of STA-6 Section 2 or Cell 5 of STA-6 Section 1. Based on a simulation period of at least thirty years, the divided configuration for the cells provided an annual phosphorous reduction of 65 to 100 kg. The reduction benefit does not appear to justify the cost of construction of the additional levees and interior water control structures.

1.0 INTRODUCTION

Substantial progress towards reducing phosphorus levels discharged into the Everglades Protection Area (EPA) has been made by the State of Florida and other stakeholders. The combined performance of the BMP source control regulatory program in the Everglades Agricultural Area (EAA) and the Stormwater Treatment Areas (STAs) of the Everglades Construction Project, both mandated by the Florida's Everglades Forever Act (EFA), has exceeded expectations. Nonetheless, additional measures are necessary to ensure the discharges to the EPA meet water quality goals including compliance with the phosphorus criterion established in Rule 62-302.540. F.A.C. In response, the "Long Term Plan for Achieving Water Quality Goals" dated October 27, 2003, was developed by technical representatives of the South Florida Water Management District, (District), the Florida Department of Environmental Protection, (FDEP), the EAA Environmental Protection District, and other stakeholders for achieving compliance with the phosphorus criterion. The "Long Term Plan" contains an initial phase to be completed by December 31, 2006, and a 10-year second phase.

The detailed designs of the physical works for STA-5 Flow-way 3 and STA-6, Section 2 were substantially completed in FY 2004. The projects in the October 27, 2003 Long-Term Plan were designed to achieve compliance with the water quality standards for the EPA by December 31, 2006, based on specific assumptions and the best available information. The Long-Term Plan was submitted on December 19, 2003, to the FDEP in accordance with the EFA requirement (Section 373.4592 (10)(a). F.S.). This was part of the District's application for a permit modification needed to implement the Long-Term Plan.

<u>Background</u>

The strength of the Long-Term Plan is ***the adaptive management process*** built into its implementation. The District and other stakeholders have continued to evaluate the inflows and phosphorus loads anticipated to enter the STAs. The first comprehensive update to this data set is expected to be complete in FY2005, with a priority being placed on the STAs discharging to the Everglades Refuge. Preliminary efforts indicate that the flows and loads entering the STAs will be considerably more than were anticipated during the development of the October 27, 2003 version of the Long-Term Plan, particularly for STA-1W and STA-2. In addition, work is underway to update the STA performance projections based on updated calibration sets, including the full-scale operations of the STAs. These performance projections are also planned to be completed in FY2005.

As part of the ***adaptive implementation process*** envisioned by the Long-Term Plan, it was anticipated that further refinements to the recommended water quality improvement measures would be made at the earliest achievable dates as more scientific and engineering information was obtained. One of the key assumptions during the development of the Long-Term Plan was that Compartments B and C (see Figure 1) would be under consideration for use as part of the Everglades Agricultural Area (EAA) Storage Reservoir Project, a component of the Comprehensive Everglades Restoration Plan (CERP), through FY 2010 and for this reason should not be considered for other Everglades restoration uses until FY 2011. Subsequent to completion of the Long-Term Plan, conceptual level analyses indicated that all of the EAA Storage Reservoir Project's CERP water storage goals could be achieved by Compartment A, and that Compartments B and C would not be needed to meet the storage objectives of the EAA Storage Reservoir CERP Project. In light of the recent availability of the land in Compartments B and C, it is currently proposed to initially expand STA-2 with a new Cell 4 and to initially expand STA-5 with a new third flow-way to assist in maximizing the treatment effectiveness of the STAs in improving water quality entering the EPA. These initial expansions are proposed to be completed as soon as possible, with a target completion date for these expansions of December 31, 2006. However, that date may be optimistic in light of issues such as permitting, real estate, cultural resources, and the major construction activities being proposed.

The detailed designs of the physical works in accordance with the initial phase of the "Long Term Plan" for the Stormwater Treatment Areas STA-5 and STA-6-2 have been substantially completed. However, approximately 8,800 acres of land owned by the District that lies between STA-5 and STA-6, Section 2 (Compartment C), has been recently vacated by the U.S. Sugar Corp. and is now available for use by the District for water treatment. It is recognized the most effective means to achieve the compliance goals will be to utilize Compartment C lands and revise the "Long Term Plan". The District also desires to make use of Compartment C lands that are in excess of that needed to achieve the water quality compliance goals for additional enhancement of the treatment capabilities of the STA's as well as to improve the operational flexibility. Consequently, it is now proposed to construct additional treatment areas on the remaining acreage of Compartment C to further assist in maximizing the effectiveness of the STAs in improving water quality entering the EPA. It is further proposed to construct the structural and vegetation enhancements identified in the Long-Term Plan for STA-2 and STA-6 Section 1 after flow-through operation of the additional treatment cells begins. The revised Part 2 of the Long-Term Plan dated November 2004 has been submitted to the FDEP as part of an amended application for permit modification and was approved on December 3, 2004.

It is intended that all six of the STAs be operated to maximize the amount of water treated e.g., no bypass of the STAs should be permitted except under extreme circumstances in which the hydraulic or treatment capacity of the STAs is exceeded, or unless the demand for downstream water supply deliveries necessitates delivery of untreated water. It is further intended that the operation of the STAs not negatively impact flood protection. Ancillary uses of the STAs for purposes other than water quality improvement will be limited to uses that do not negatively impact treatment performance.

The Compartment C tract will be developed in two phases to comply with immediate and long-term goals. In this report, the first phase is referred to as the Initial Configuration and the second phase is referred to as the Build Out Configuration. The Initial Configuration phase includes the construction of additional treatment cells designated STA-5 Flow-way 3 and STA-6 Section 2. A map of the STA-5/6 areas is shown on Figure 2. Figure 3 presents schematics of proposed structures to be evaluated for the Initial Configuration and Figure 4 presents schematics of proposed structures to be evaluated for the completed Build Out Configuration. These figures are based on the schematic for the STA-5 enhancements shown in Figure 2.15 from the Long Term Plan.

2.0 SCOPE OF WORK

The following scope of work was performed as part of this final Preliminary Modeling deliverable.

Hydraulic and Hydrological Modeling: Performed the analysis of flows through the treatment areas and determined the proper sizing of the hydraulic structures for the new flow-ways. The District provided a general plan for the new flow-ways as well as the build-out of Compartment C. The plans included the proposed hydraulic structures, sized and with invert elevations defined as a starting modeling scenario.

The hydrologic and hydraulic analyses performed consisted of site-specific and sub-regional models to determine the backwater effects of the project on the basin's flood protection in conjunction with the operation of existing treatment works. The objective was to maintain the regional flood discharges with the addition of the project. The new flow-ways were analyzed as gravity systems, and hydraulic structures were sized for this condition. The future build-out components were analyzed both as gravity systems and as pumped systems with the addition of a discharge pump station. The hydraulic structure design is in accordance with the District's standard computational methods.

The following flow conditions were analyzed:

1. Design Normal Peak Flows (Design)
2. Standard Project Flood Condition
    a. When the Miami Canal is not in flood mode (No Flood SPF)
    b. When the Miami Canal is in flood mode (Flood SPF)

The flood condition on the Miami Canal limits the flow that can be discharged through the STA-5 Discharge Canal. The total Miami Canal flow is restricted by the combined pumping capacity of the S-8 and G-404 pump stations.

The STA-5/6 system will be expanded in phases. The Initial Configuration adds STA-5 Flow-way 3 and STA-6, Section 2 to the existing flow-ways. The Build-Out Configuration adds STA-5 Flow-ways 4 and 5 along with STA-6, Cell 4. Components of these two configurations are evaluated in the five modeling scenarios below.

Flows from each basin for each flow condition were provided by the District and are summarized in Table 2.1.

Table 2.1 – Tabulated Listing of Flow Conditions

Summary of Anticipated Flow Conditions (all flows in cfs)	Flow	Total STA-5 Inflows	C139 Diversion (G406)	Total STA-6 Inflows (Note 1)	Diversion (G407) (Note 2)	Comments
2007-2008 (1/2007 - 12/2008)						
C-139 Basin						Three flow-ways for STA-5; discharge to Miami Canal
Modified Design Condition	2,096	1,790	306	0 - 306		2,096 was max. flow in 1965 -1995 period; max through STA-5 kept at 1,790 cfs per 1997 design
Standard Project Flood	3,440					SPF estimated in 1997 design for STA-5
Miami Canal not in flood mode		2,510	930	0-930		STA-5 inflow value from 1997 design
Miami Canal in flood mode		1,080	2,360	0-1,228		STA-5 inflow value from 1997 design
Unit 2						
Design	300			300		Existing capacity of G-600
Standard Project Flood	300			300		Existing capacity of G-600
C-139 Annex						
Design	452			452		Maximum permitted discharge capacity
Standard Project Flood	452			452		Maximum permitted discharge capacity
Total flows						
Modified Design Condition	2,848	1,790	306	752 - 1,058	0 - 306	
Standard Project Flood	4,192					
Miami Canal not in flood mode		2,510	930	752 - 1,682	0 - 930	
Miami Canal in flood mode		1,080	2,360	752 - 1,980	1,123 - 2,360	
Post-2008 (1/2009 -)						
C-139 Basin						
Modified Design Condition	2,096	2,096	0	0	0	Flows to Miami Canal maintained at or below pre-2008 levels; balance to L-4
Standard Project Flood	3,440					Flows to Miami Canal maintained at or below pre-2008 levels; balance to L-4
Miami Canal not in flood mode		3,440	0	0	0	
Miami Canal in flood mode		3,440	0	0	0	
Unit 2						
Design	0			0		G600 taken out of commision
Standard Project Flood	0			0		G600 taken out of commision
C-139 Annex						
Design	452			452		Maximum permitted discharge capacity
Standard Project Flood	452			452		Maximum permitted discharge capacity
Total flows						
Modified Design Condition	2,548	2,096	0	452	0	
Standard Project Flood	3,892					
Miami Canal not in flood mode		3,440	0	452	0	
Miami Canal in flood mode		3,440	0	452	0	

Note 1. Optimal STA-6 inflow to be determined through analysis; capacity estimated in 1997 design was 1,980 cfs.
Note 2. Optimal G407 capacity to be determined through analysis.

The following modeling scenarios were analyzed:

(1) Base Case: STA-6 Sections 1 and 2 Inflows: C-139 Annex Flows, Unit 2 Flows; and C-139 diversion. This simulation scenario shall analyze the STA-6 Sections 1 and 2 (including cells 3 and 5 as modified with new inflow structures) with the addition of a new Section 2 immediately to the north. Section 2 shall be 1.5 miles wide (north to south) and shall extend from the USSC Unit 2 main canal east to the Rotenberger Track. All flow shall be from west to east and discharge through gated control structures to an extension of the discharge canal that parallels the Rotenberger Track. STA-6 Section 1, Cells 3 and 5 shall be modified to connect directly to the L-3 borrow canal by means of three (3) operable box culverts through the L-3 levee. Inflow to Section 2 shall be through gated control structures supplied by an inflow canal connected to the L-3 borrow canal immediately south of the USSC G600 pump station. STA-6 Section 1, Cells 3 and 5 currently each have three (3) weir box culverts for discharge and shall remain. The discharge canal shall connect to the L-4 Borrow Canal with eventual discharge to Water Conservation WCA-3A. A bypass structure G-407A shall be constructed in the L-3 borrow canal just north of the Oil Well Bridge to prevent flow to the south unless under emergency conditions or when a portion of the C-139 runoff cannot be treated in STA-5 (estimated maximum of 2,360 cfs).

The analysis considered inflow to STA-6 from the USSC C-139 annex from a single pump station with maximum capacity of 452 cfs. The currently permitted stage-discharge relationship was used for this pumped facility. The existing G600 pump station was limited to 300 cfs. The pump station as it presently exists is in some disrepair. A 2004 fire destroyed one pump and a second pump is being rebuilt leaving only 3 potentially operable pumps which could supply the permitted discharge. The analysis shall also consider the conveyance of C-139 Basin runoff through the L-3 borrow canal that is diverted away from STA-5 due to hydraulic constraints in the STA-5/S-8 system, up to a maximum of 2,360 cfs. The total treatment area of STA-6 shall be dedicated to the C-139 Annex pumped runoff, the discharge from USSC Unit 2 and runoff diverted from the C-139 Basin which should be divided proportionately between the three flow paths.

This is the primary design scenario for normal operation and will therefore be the analysis used for the sizing of the conveyance facilities. The analysis determined the following:

- Optimum size of the four (4) inflow gravity structures to Section 2 from the L-3 borrow canal at maximum discharge capacity of the USSC pumps.
- Optimum size of the four (4) gravity discharge structures from Section 2 to the discharge canal.
- Optimum size of the three (3) gravity inflow structures to Section 1, cells 3 and 5 from the L-3 borrow canal.
- Optimum size of the G-407A divide structure to ensure that stages at the G-406 structure do not exceed existing conditions.
- Maximum stage in the L-3 borrow canal given the selected structures through the system.
- Maximum stage in the treatment cells 3, 5, and Section 2 at the HW to the discharge structures.

It is noted that the number of above Intake and Discharge structures for Section 2 originally requested to be evaluated were considered excessive compared to those installed at STA –5 Flow Ways 1 and 2. This final submittal evaluates a reduction in the number of Intake and Discharge Structures requested to be initially evaluated as a value engineering alternative.

An engineering opinion of probable cost has been prepared for the new or modified structures and earthwork identified in the Base Case. This cost estimate includes capital costs and projected annual operation and maintenance costs.

(2) Mixed Discharge Alternative: This alternative includes STA-6; C-139 Annex Flows in addition to 300 cfs from the USSC Unit 2 pump station and a 2,360 cfs discharge from the C-139 basin through the L-3 borrow canal G-406 divide structure.

Using the STA-6 facilities as described in the Base Case above, this scenario evaluates the hydraulic conditions associated with a mixed gravity and pump discharge from the C-139 Annex instead of the pump station contained in the Base Case.

The analysis considered inflow to STA-6 from the USSC C-139 Annex holding ponds from two (2) gravity structures and a supplemental pump station with a combined maximum capacity of 452 cfs. URS used the currently permitted stage-discharge relationship for this mixed facility. The existing G600 pump station shall be limited to 300 cfs. The analysis also considered the conveyance of C-139 Basin runoff through the L-3 borrow canal that is diverted away from STA-5 due to hydraulic constraints in the STA-5/S-8 system, or approximately 2,360 cfs. The total treatment area of STA-6 shall be dedicated to the C-139 annex runoff and the discharge from Unit 2, and runoff diverted from the C-139 Basin, which should be divided proportionately between the three flow paths.

The analysis determined the following:

- Optimum size of the two (2) gravity control structures and supplemental pump station from the USSC holding ponds to the L-3 borrow canal at a total discharge of 452 cfs and the maximum stage in the holding ponds.
- Optimum size of the four (4) inflow gravity structures to Section 2 from the L-3 borrow canal at maximum discharge capacity of the USSC pumps.
- Optimum size of the four (4) gravity discharge structures from Section 2 to the discharge canal.
- Optimum size of the three (3) gravity inflow structures to STA – 6 Section 1, cells 3 and 5 from the L-3 borrow canal.
- Optimum size of the G-407A divide structure to ensure that stages at the G-406 structure do not exceed existing conditions.
- Maximum stage in the L-3 borrow canal given the selected structures through the system.

- Maximum stage in the treatment cells of STA-6 Section 1, Cells 3 and 5, and STA-6 Section 2 at the HW to the discharge structures.

It is noted that the number of above Intake and Discharge structures for Section 2 originally requested to be evaluated, were considered excessive compared to those installed for STA– 5 Flow-ways 1 and 2. This submittal evaluates a reduction in the number of Intake and Discharge Structures as a value engineering alternative.

An engineering opinion of probable cost has been prepared for the new or modified structures and earthwork identified in the Mixed Discharge Alternative scenario. This cost estimate includes capital costs and projected annual operation and maintenance costs.

Upon completion of the analysis of the two scenarios outlined above, URS presented this information to the District in the form of a report. The District's Project Manager was then to decide which scenario, (1) Base Case or (2) Mixed Discharge Alternative, should be utilized in the subsequent scenarios.

(3) STA-6 Build-out Condition: This simulation scenario analyzed the Build-out configuration with only the C-139 Annex inflow. The build-out configuration includes the addition of an additional cell immediately west of Section 2 and extending to the L-3 borrow canal. Flow to the expanded/combined STA-6 Section 4/2 Flow-way shall be from the L-3 borrow canal through four (4) gated control structures. The analysis considered inflow from the C-139 Annex based on the configuration determined by the District.

The analysis determined the following:

- Optimum size of the four (4) gravity structures into Section 4 from the L-3 borrow canal given a maximum discharge capacity from the C-139 Annex of (452 cfs).
- Maximum stage in the L-3 borrow canal given the selected gravity structures through the system.
- Maximum stage in the treatment cell at the HW to the gated discharge structures.

It is noted that the number of above Intake and Discharge structures for Section 4 originally requested to be evaluated, were considered excessive compared to those installed for STA–5 Flow-ways 1 and 2. This submittal evaluates a reduction in the number of Intake and Discharge Structures as a value engineering alternative.

(4) STA-5 Flow-way 3 Condition: This modeling scenario analyzed the addition of a 1-mile wide Flow-way 3 immediately south and parallel to the existing STA-5. The flow shall be from west to east with inflow from the L-2/L-3 borrow canals through gated control structures. The flow-way was modeled as two cells in series: an emergent vegetation treatment cell followed by a submerged aquatic vegetation treatment cell. The flow shall discharge from this flow-way to a discharge canal parallel to the Rotenberger Tract boundary. The discharge shall be to the existing STA-5 outfall canal which connects to the Miami canal. The existing STA-5 shall be modified by others with the conversion of the interior weir box culverts, G343A through H, to gated, remotely operated control structures. The simulation considered the historical discharge from the C-139 basin via the L-2/L-3 borrow canals, with all control structure inflow and discharge canal structures fully open.

The analysis determined the following:

- Optimum number and size of the gravity inflow structures to the new flow-way addition to ensure that stages at the G-406 structure do not exceed existing conditions. The sizing shall optimize the flow split with the existing two STA-5 flow paths and the new flow-way.
- Optimum number and size of the gravity discharge structures from the new flow-way 3 addition to the new discharge canal.
- Optimum size of the discharge canal from the new flow-way 3 to its connection to the existing STA-5 discharge canal. Note this portion of the canal will need to convey flows from the build-out of Compartment C. Therefore the sizing shall be based on condition (5) described below.
- Maximum stage in the L-3 borrow canal under projected project storm conditions.
- Maximum stage in the cells at the HW to the gated discharge structures.

It is noted that the number of above Intake, Internal and Discharge structures for STA–5 Flow-way 3 originally requested to be evaluated, were considered excessive compared to those installed for STA–5 Flow-ways 1 and 2. This submittal evaluates a reduction in the number of Intake, Internal and Discharge Structures as a value engineering alternative.

(5) STA-5 Build-out: This modeling scenario analyzed the configuration from scenario (4) above with the addition of two (2) additional flow-ways utilizing the remaining Compartment C lands to the STA-6 build-out. The northern flow-way shall extend one mile from north to south, while the southern flow-way shall extend the remaining 1.5 miles south to the STA-6 build-out. Flows shall be from west to east with inflow from the L-3 borrow canal and discharge to extension of the discharge canal along the Rotenberger Tract. Each flow-way was modeled as two cells in series, an emergent vegetation treatment cell followed by a submerged aquatic vegetation treatment cell. The discharge canal shall be connected to the STA-6 discharge canal and shall have a divide structure for control of flows north or south. Each cell shall three (3) inflow control structures and three (3) discharge structures.

The analysis determined the following:

- Optimum size of the gravity inflow structures (total of 6) to the new cells additions. The sizing shall optimize the flow split with the configuration described in scenario (4) above.
- Optimum size of the gravity discharge structures (total of 6) from the new cell addition to the new discharge canal.
- Optimum size of the discharge canal from the new cells to its connection to the existing STA-5 Outfall canal.
- Maximum stage in the L-3 borrow canal under projected project storm conditions.
- Maximum stage at the HW to the gated discharge structures.

It is noted that the number of above Intake, Internal and Discharge structures originally requested to be evaluated, were considered excessive compared to those installed for STA–5 Flow-ways 1 and 2. This submittal evaluates a reduction in the number of Intake and Discharge Structures as a value engineering alternative.

If necessary, to prevent a reduction in upstream flood protection, the analysis was to consider an outflow pump station located at the confluence of STA-6 discharge canal and the L-4 Borrow Canal.

This final Preliminary Hydraulic Modeling report documents the Hydraulic Design and Water Quality Modeling for the new treatment flow-ways in Compartment C and STA –6 Section 2 including hydraulics and hydrology. All water surface elevation and topographic data provided in this report are referenced to the National Geodetic Vertical Datum (NGVD) of 1929.

3.0 STA MODELING INFLOWS

Inflow to STA-5 and ST-A6 is from two hydrologic basins as shown on Figure 2. The C-139 Basin lies to the North and west of the existing STA-5 cells. The C-139 Annex Basin lies generally to the north and west of the existing STA-6 cells. The long-term routing of stormwater from these two basins primarily directs the flow from the C-139 Basin to the STA-5 cells while the flow from the C-139 Annex Basin is directed to the STA-6 cells.

Inflow to the STA-5 and 6 systems depend on the stage and flow rate of the L-3 canal system as well as rainfall captured in the C-139, C-139 Annex, and Compartment C basins. Inflow from the C-139 Basin has been documented in previous design reports for STA-5 and STA-6, *Stormwater Treatment Area No.5 Assessment of Operational Impacts*, (Burns and McDonnell, November 1999) and *STA-6, Section 2 Hydraulic Modeling*, (Burns and McDonnell, June 9, 2004). Those reports cite analyses of historic flow records from the District's DBHYDRO database. The maximum reported flowrate from the C-139 Basin for the period of record was 2,096 cubic feet per second (cfs). This flow was adopted by the District as the Design Flow from the C-139 Basin. The District also established the Standard Project Flood (SPF) flow rate from the C-139 Basin as 3,440 cfs. These and other system flow rates are shown in Table 2.1.

Inflow from the C-139 Annex Basin is via a proposed pump station and reservoir system constructed by USSC. Three USSC-operated pump stations discharge into the USSC reservoir. Discharge from the USSC reservoir is limited to 452 cfs by an existing discharge permit. The Design and SPF flow rates for the C-139 Annex Basin were therefore taken as 452 cfs.

Because large portions of Compartment C are undeveloped prior to system build out, stormwater from undeveloped portions of that tract will be directed to STA-6 during the period after the Initial Configuration project is completed and prior to Build Out Configuration completion. During this interim period, Compartment C stormwater will be directed to the STA-6 cells. Discharge from the Compartment C tract is through the existing G600 pump station. That pump station contains five 100-cfs pumps. A fire at the pump station has left only three of the pumps operable. The Design and SPF flow rates from the Compartment C tract are therefore 300 cfs.

Prior to the completion of the Build Out Configuration, inflows to STA-6 will be from a combination of C-139 Annex, C-139, and Compartment C basin sources. The only inflow to the STA-5 cells will be the C-139 Basin. Prior to the Build Out project completion, a portion of the C-139 Basin flow may be by-passed to either the STA-6 cells or directly to Water Conservation Area 3 (WCA-3A) via the existing G-406 diversion structure and the L-3 Canal. Flow by-pass to WCA-3A is expected to occur only during extreme (SPF) events. The SPF event is estimated to have a recurrence interval of 100 years or longer.

Inflow quantities for the STA-6 cells were distributed based on their relative treatment areas. The flow distributions for the cells are shown in Table 3.1. These percentages differ slightly from each cell's overall area to provide an adjustment for "effective" treatment area. The STA-6 Section 2 distribution is 2 percent larger while the other cells are each 1 percent less than their area ratios. In the Build Out Configuration, the flow distribution to all STA-5 and STA-6 Flow-ways were estimated based on their relative treatment areas. The goal was to create an equivalent hydraulic loading rate across the treatment areas.

Table 3.1 STA-6 Flow Distributions

STA Name	Area Ratio (%)	Recommended Flow Ratio (%)
STA-6 Flow-way 2	61	63
STA-6 Flow-way 3	11	10
STA-6 Flow-way 5	28	27

Schematics of the flows through the STA-5/6 system are shown in Figures 5 and 6. Figure 5 shows the flows under the Initial Configuration. In that configuration, STA-5 Flow-way 3 is added to STA-5 system and (STA-6 Section 2) is added to STA-6 system.

The STA-5 discharge will flow to south to the S-8 and G-404 pump stations where it will be discharged to WCA-3A. Return flow from STA-3/4 will re-enter the Miami canal via the L5 Canal adjacent to the S-8 pump station. The STA-3/4 return flow is reported at up to 3,700 cfs. The combined pumping capacity of the S-8 and G-404 pump stations is reported by the District as about 4,780 cfs, leaving 1,080 cfs available for STA-5 discharge flow. This flow limit was imposed for the STA-5 Discharge Canal SPF when the Miami Canal is under flood condition. The STA-5 discharge flow increases to 2,510 cfs when the Miami Canal is not under flood condition (Burns and McDonnell, 2004).

The STA-6, Section 2 Hydraulic Modeling report indicated the outflow through the STA-5 Discharge Canal must be limited to 1,080 cfs when the Miami Canal is in Flood Condition (Burns and McDonnell, 2004). When the Miami Canal is not in Flood Condition, the STA-5 Discharge Canal flow may be up to 2,510 cfs for the Initial Configuration. Because their treatment areas are similar, flow through the Initial Configuration of STA-5 is evenly split between the three treatment cells under all Design and SPF flow conditions. Hydraulic limitations within the treatment cell and canal system also affect the flow distributions.

Although flow limitations through STA-5 are imposed in the Build Out Configuration, Figure 6 shows the flow through the STA-5 Discharge Canal will not exceed 1,500 cfs under the Design or SPF conditions.

The G-406 diversion structure will be retained in the Build Out Configuration to provide a metering and monitoring point for C-139 Basin flows. The L-3 Canal becomes a common inflow conduit for all STA-5/6 cells in the Build Out Configuration. Flows from the C-139 and C-139 Annex basins will be commingled in the L-3 Canal. The G-407A diversion structure will divert flow from both the C-139 Basin and C-139 Annex Basin to the treatment cells.

The increased flow capacity of the treatment cells in the Build Out configuration should accommodate all Design and SPF flows. Flow of untreated water to WCA-3A may occur during the Initial Configuration under SPF flow when the Miami Canal is in Flood condition. Figure 6 shows the distribution of flows through the treatment cells under the Build Out configuration. In the Build Out configuration, all the proposed and existing treatment cells are in operation.

The G-406 structure is located on the L-3 Canal for the purpose of diverting flow through the existing STA-5 treatment system. The structure was designed to by-pass excess C-139 Basin flows to WCA-3A when the L-3 Canal stage at Deer Fence Canal exceeds 17 feet NGVD. Figure 5 shows the by-pass flows through G-406 under the various flow conditions.

By-passed flows will be prevented from flowing directly to WCA-3A by a proposed G-407A diversion structure. That structure will be similar to the existing G-406 structure. All the by-passed flow will be directed to the STA-6 cells when possible. Under the Flood SPF condition for the Initial Configuration, the C-139 Basin flow is by-passed to WCA-3A at 2,000 cfs. The Flood SPF event is an exceptional event and would not be expected to normally occur.

The distribution of flow through the treatment system is primarily based on each cell's treatment area, but the limitations imposed by the STA-5 Discharge Canal flow limits have altered the distribution somewhat. Hydraulic limitations within the treatment cell and canal system also affect the flow distributions.

Water quality modeling was performed in this study to estimate the phosphorus removal of the proposed STA's. The modeling used historic input datasets that incorporate daily inputs for flow, phosphorus concentration, rainfall, and evapotranspiration from either the C-139 Basin or C-139 Annex Basin. The STA-5 modeling utilizes the C-139 Basin dataset while the STA-6 modeling utilizes the C-139 Annex Basin dataset. Both datasets span the time period from January 1, 1965 to April 30, 2003. The datasets were provided by the District for this modeling effort. URS modified the datasets as needed to reflect the transfer of some C-139 flow to the STA-6 system.

4.0 CANAL FLOWS

The flow in canals directly or indirectly connected to the treatment cells affects the flow capacities and water surface elevations in the treatment system. The flow assumptions for these canals are described below.

The Miami Canal is the predominant drainage canal for this region. The S-8 pump station controls the Miami Canal water surface elevation draining the Everglades Agricultural Area north of the pump station. The STA-3/4 treatment system will be used to treat water from the Miami Canal. During operation, STA-3/4 will draw up to 3,700 cfs from the Miami Canal at a point near the G-200 pump station (Burns and McDonnell, 2004). Figure 2 shows the location of G-200 on the Miami Canal. A diversion structure (G-373) has been constructed on the Miami Canal at that location to direct the Miami Canal flow to the STA3/4 system. A separate STA-5 Outflow Canal has been constructed parallel to the Miami Canal to convey the STA-5 Discharge Canal flow to a point South of the diversion structure. The remainder of the Miami Canal running south to the S-8 pump station was assumed to carry only STA-5 discharge.

Runoff from the Rotenberger Tract is reflected in the flow records of the four G-402 structures (A through D). Review of the four G-402 structure flow records from DBHYDRO showed that the runoff contribution from the Rotenberger Tract may be about 100 cfs from each of the structures. Given the limited S-8/G-404 pumping capacity, any flow contribution from the Rotenberger Tract may be directly borne as a flow reduction through the STA-5 system and potentially more by-pass to WCA-3A via G-407A. For the Design and No Flood SPF conditions, the 100 cfs contributions from each of the G-402 structures were modeled. For the Flood SPF condition, the pump stations do not have excess capacity. Therefore, the G-402 inflows were not modeled as this would create less flow and less headloss through the Miami and STA-5 Canals.

The L-4 Canal conveys flow from the Miami Canal to WCA-3A. The G-404 pump station discharges into the L-4 Canal when additional discharge from the Miami Canal is needed. The G-404 pump station has a flow capacity of 570 cfs. Flow in the L-4 Canal is primarily from the G-404 operation, but flow through the G-357 control structure is possible when the L-4 Canal water surface elevation (WSE) is low enough. For this hydraulic study, the flow into L-4 was assumed to be 570 cfs under SPF conditions and one-half that flowrate (285 cfs) during the Design condition. This approach is identical to the approved approach used for the STA-6, Section 2 Hydraulic Modeling report (Burns and McDonnell, 2004).

The L-28 Canal provides another flow outlet from the project area. The L-28 Canal parallels the L-3 Extension Canal, but turns and extends farther south. The L-28 Canal is separated from WCA-3A by an earthen levee. The S140 pump station discharges water from L-28 into WCA-3A at a rate of up to 1,300 cfs. Inflow to the L-28 Canal is regulated by the G-89 control structure. Flow through the G-89 structure was assumed to be zero in this study. This directs all the flow to the L-4/L-3 Extension Canal system and WCA-3A producing a higher WSE in the L-4 Canal.

5.0 MODELING SCENARIOS

Five modeling scenarios are included in this project deliverable. The base scenario evaluated the hydraulics for the operation of STA-6 Sections 1 and 2 with a pumped discharge from the C-139 Annex reservoir. Sections 1 and 2 cells are shown on Figure 3. The second scenario examined a mixed flow from the C-139 Annex consisting of either gravity flow from the C-139 Annex reservoir or pumped flow. The third scenario examined the STA-6 Build Out condition and the number and sizing of its hydraulic structures. The forth scenario examined the STA-5 Flow-way 3 and its hydraulic structures. The final scenario examined the STA-5 Build Out and the number and sizing of its hydraulic structures.

The first project scenarios focuses on the Initial Configuration shown on Figure 3 of the STA-6 treatment system consisting of the 2 sections and three cells mentioned above. STA-6 Section 2 is a proposed cell, while STA-6 Section 1, Cell-3 and STA-6 Section 1, Cell 5 are existing. Inflow to STA-6 Section 2 will be via a new inflow canal beginning at the existing G600 pump station. Three new gated inflow structures will convey flow from the inlet canal to STA-6 Section 2. Three new gated outlet structures will convey the flow from STA-6 Section 2 to the STA-6 Discharge Canal. The existing STA-6 Discharge Canal will be extended to parallel the STA-6 Section 2 Cell. Three new gated inlet structures will be constructed for the STA-6 Section 1, Cell-3 and STA-6 Section 1, Cell 5 to replace the existing overflow weir inlets. A new diversion structure (G-407A) will be constructed in the L-3 Canal to direct flow to STA-6 and limit discharge from the L-3 canal to WCA-3A.

The second scenario includes the evaluation of two new gravity discharge structures from the C-139 Annex reservoir. This case also includes a supplemental pump station to discharge C-139 Annex flow to the L-3 canal when the canal experiences high stages.

The Build Out condition for the STA-6 system was examined in the third scenario.

The proposed STA-5 Flow-way 3 cell is unaffected by the scenarios presented above. The evaluation of the STA-5 Flow-way 3 system was performed as required in Scenario 4. Flow through the STA-5 system is entirely from the C-139 Basin. The STA-5 system was evaluated in coordination with the STA-6 system.

The buildout of the STA-5 system was evaluated in Scenario 5. The complete buildout configuration of STA-5 and STA-6 is shown on Figure 4.

5.1 Hydrologic Modeling

The hydrology for the project area is well defined by previous engineering studies (Burns and McDonnell, March 1999). The Design and Standard Project Flood (SPF) flows from the C-139 Basin were previously approved and adopted by the District and used in this study. The Design flow (2,096 cfs) was taken in the previous study as the highest observed flow from the C-139 Basin. The SPF flow (3,440 cfs) was a factored estimate of the design flow. A tabulation of flows through the project area is provided in Figures 5 and 6. The figures show the proposed flow routing through the system.

Hydrologic modeling of the treatment cells was performed to assess the outflow performance of the treatment cell outfall structures and the subsequent WSE rise during the storm event. The outflow performance was evaluated using the Standard Project Storm (SPS) event. The SPS is defined as a 21.6-inch, 24-hour duration rainfall event. The hourly incremental rainfall for the SPS event was taken from the Detailed Design Report Stormwater Treatment Area No. 6 (Burns and McDonnell, March 1997).

The hydrologic modeling was performed using HEC-HMS modeling software produced by the U.S. Army Corps of Engineers (USACOE). The purpose of the analyses was to evaluate the peak stages within the proposed cells and the peak flow through the hydraulic structures. The interaction of the storage and outflow capacity of the structures helped determine the proper sizing of the structures.

STA-5 Flow-way 3 - For the HEC-HMS modeling, the flow-way cells were modeled as a linked watershed models. Each model consists of a pair of runoff models for upstream and downstream cells of the flow-ways. The runoff models represent the open-catchment of rainfall within the cells. The runoff models were configured to produce 100-percent of the rainfall as runoff. The flow from each runoff model was directed to individual reservoir models. Each cell's reservoir model includes the cell's calculated stage/storage/outflow relationship. Each cell's storage was based on its estimated water surface area. The outflow capacity was based on the structure rating tables developed for each cell's structures (Appendix B). Two 10' by 8' gated inlet box culverts were determined to be suitable for STA-5 Flow-way cells 3A and 4A and two 10' by 10' gated discharge box culverts were determined to be suitable for STA-5 Flow-way cells 3B and 4B. The structure sizes stayed the same for STA-5 Flow-way 5, but the number of structures increased to three at each cell outlet.

STA-6 Section 2 and 4 Buildout - The STA-6 Section 2 cell was modeled similarly to the STA-5 Flow-way systems. Two 10' by 8' gated inlet box culverts were determined to be suitable for STA-6 Section 4 inflow from the L-3 canal. Since STA-6 Section 2 will eventually receive inflow from STA-6 Section 4 in the Build Out, a linked watershed model was used. The outflow capacity of the STA-6 Section 4 cell requires that three 8' by 8' gated box culverts will be used. The outflow capacity of STA-6 Section 2 requires the use of three 10' by 8' gated box culverts.

The existing STA-6 Section 1, Cell-3 and STA-6 Section 1, Cell 5 cells were not evaluated since their outflow structures were not undergoing redesign as part of this project. These structure's hydraulic capacity and headloss estimates were calculated for use in the cell hydraulic modeling and water surface profile calculations.

5.2 Hydraulic Modeling

5.2.1 One-Dimensional Modeling

Hydraulic modeling of the canal systems affecting the STA area was conducted to assess the water surface elevations under the various operating conditions and configurations. HEC-RAS modeling software produced by the USACE was used to develop and evaluate the water surface profiles in the canals. The work included the development of five HEC-RAS models. The models are listed below.

- L-3 Canal model
- Miami Canal model
- STA-5 Discharge Canal model
- STA-6 Discharge Canal model
- Southern model

A calibrated HEC-RAS model of the L-3 Canal was developed during previous design work for STA-5 (Burns and McDonnell, March 1999). That model was adopted for this analysis. That model was developed from canal cross-sections provided by the District and from flow and level information taken from DBHYDRO. The calibration indicated a Manning's "n" of 0.0229 for the main channel produced the best agreement between observed and modeled conditions. Typical Manning's "n" textbook values for straight, earthen channels range from 0.022 to 0.030 (Chow, 1959). The G-406 structure was apparently not in place when the L-3 model calibration was performed. The G-406 diversion structure along with the G-407A diversion structure were incorporated into the L-3 model by URS.

Cross-sections for the Miami Canal model were taken partially from a previous engineering report for the STA3/4 design (Burns and McDonnell, April 1996). URS calibrated the Miami Canal model using flows and water surface elevations reported by the District's DBHYDRO database. The headwater at S-8 pump station, the tailwater at the four G-402 structures and the headwater at the G-200 pump station provided the water surface information for calibration. The locations of these structures and monitoring locations are shown on Figure 2. The Miami Canal flow was taken as the sum of the S-8 and G-404 pump station flows and the G-357 flow. The model was able to produce relatively good comparisons to measured stages under differing flow conditions using a Manning's "n" of 0.029 in the main channel. The calibration results for the Miami Canal are shown in Appendix A.

The STA-5 Outfall Canal parallels the Miami Canal from the confluence of the Miami Canal and the STA-5 Discharge Canal to past the G-373 diversion structure. The STA-5 Outfall Canal routes the STA-5 flow to a point below the G-373 structure to prevent commingling of the untreated Miami Canal flow and the treated STA-5 flow. The STA-5 Outfall Canal is reportedly under construction. The cross-section of the STA-5 Outfall Canal was assumed to be similar to the proposed section for the STA-5 Discharge Canal. The STA-5 Outfall Canal sections were appended northward of the G-373 structure in the

Miami Canal model. The Manning's 'n' values found for the Miami Canal calibration were continued for the STA-5 Outfall Canal component.

The STA-5 Discharge Canal model was developed using the as-built cross-sections for the canal provided by the District (Burns and McDonnell, September 1997). The flow through the canal was taken as the sum of the G-344 (outlet structure) discharges from the STA-5 flow-ways reported by DBHYDRO. The tailwater conditions reported for the G-344 structures were used for the upstream water surface data in the STA-5 Discharge Canal. Since a gaging station is not located within the STA-5 Discharge Canal, the tailwater for the G-402D structure data was used to estimate the water surface elevation at the terminus of the canal. The G-402D structure is located approximately 1/3 mile downstream from the confluence of the STA-5 Discharge Canal and the Miami Canal. The difference in water surface elevation between the confluence and the gaging station was calculated as less than 0.01 foot under typical flow conditions. Using the calibration data, the model was able to produce relatively good comparisons to measured stages under differing flow conditions using a Manning's "n" of 0.025 for the main channel. The calibration results for the STA-5 Discharge Canal are shown in Appendix A. An additional consideration is the G410 pump station which draws water from the STA-5 Discharge Canal for discharge into the Rotenberger Tract for hydrologic restoration. All the calibration events were taken from periods when the G410 pump station was not operating.

The STA-6 Discharge Canal model is composed of the existing STA-6 Discharge Canal and its future extension to the north. The cross-sections for the STA-6 Discharge Canal were taken from the Stormwater Treatment Area No. 6 construction drawings (Burns and McDonnell, December 1996). The cross-section of the existing canal was evaluated using the maximum predicted flow for the canal shown on Figures 5 and 6 (2,812 cfs). The existing cross-section for the STA-6 Discharge Canal (2.5:1 side slopes, 34' bottom width, 0' NGVD bottom elevation) was found to be adequate when canal's water surface elevation was greater than 13 ft NGVD. But, under the maximum predicted flow, the flow velocity exceeds 2.5 feet per second near the canal terminus when the canal water surface elevation is pumped down to less than 13 ft NGVD.

To produce the intended treatment cell flows, the STA-6 Discharge Canal will need to be pumped down to about 12' NGVD in the Build Out Configuration. To alleviate the excess velocity created by the lower WSE, an enlarged canal cross-section was employed. The bottom width was increased to 40 feet and the bottom elevation was lowered to –2 feet NGVD. This canal expansion will not be required for the Initial Configuration, but can be implemented at that time.

Since a significant portion of the STA-6 Discharge Canal will be constructed in the future and there is limited flow and level data for the existing canal, calibration was not performed. A typical Manning's 'n' value of 0.025 was used for the main channel in this model. The STA-6 Discharge Canal modeling assumed that the G-607 culvert structure is replaced with a free-flowing bridge or structure.

The STA-6 Discharge Canal discharges into a system of canals flowing into WCA-3A. This system of canals was integrated into a model of this southern flow terminus. The Southern model includes the L-3 Extension Canal, L-4 Canal, L-4 Gap, and the outflow through WCA-3A. The model includes a branched flow path scheme to mimic the dual flow paths provided by the parallel L-3 Extension and L-4 canals.

The WCA-3A swamp was represented in the model as a broad floodway. Ground surface elevations for this floodway were obtained from a District GIS map (SFWMD, 1990). The cross-sections for the canals in this model were taken from Corp of Engineers drawings and surveys provided by the District (USACE, 1957, 1964, 1977). Model calibration was accomplished by using multiple gauging records provided by DBHYDRO. WSE data from G-155, 3A-NW, and S-339 were used for the calibration. The G-155 tail water, located in the L-3 Extension Canal near the south end of STA-6, was used as the upstream WSE for the model calibration. A review of other gages in the WCA-3A area indicated that the Miami Canal water level has a strong effect on the flow through WCA-3A. Water surface elevation in the Miami Canal south of S-8 is controlled by the S-339 control structure. The headwater at the S-339 structure was found to be a reliable indicator of the tail water elevation for the WCA-3A flow path. The 3A-NW gage is located approximately midway between the Miami Canal and the terminus of the L-3 Extension Canal. It provided a mid-point calibration. The flow through WCA-3A was estimated as being the sum of the G-404 flow, STA-6 flow, and L-3 flow. Flows and levels from DBHYDRO from selected periods were used to calibrate the model.

The head loss through WCA-3A was found to be the primary influence on water surface elevations in the L-3/L-4 canals. The calibration results were relatively good when a depth-weighted Manning's "n" was used. The selected Manning's n distribution was very similar to that used for the STA two-dimensional modeling. Figure 7 shows the depth-weighted Manning's "n" used for the modeling. The calibration results for this model are provided in Appendix A. This model has some uncertainty due to the complexity and size of the model and limited calibration points. This model has an effect on the modeling results for the project's Initial Configuration. This configuration is dependent upon the WSE at the south end of the project area and its effects upstream. The model has little effect on the STA5/6 system WSEs when the discharge pump station is in operation after the project build out. The model's effect on the L-3 Canal WSE is also limited after build out since by-pass flows through the G-407A structure are not anticipated.

An analysis of the calibration results for the Southern, Miami Canal, and STA-5 Discharge Canal models is provided in Appendix A. Graphs of the Goodness of Fit for the observed and modeled data are provided.

Water surface profiles for the system were compiled using the results of the 1-D modeling, 2-D modeling, and hydraulic structure head loss. Downstream WSEs for the 1-D models are shown in Tables 5.1 and 5.2. Where models are consecutively linked, the WSEs calculated at the upstream end of the downstream model were used as the downstream WSEs for the upstream model.

Table 5.1
Initial Configuration 1-D Modeling Boundary Conditions

Model Name	Flow Condition	Downstream WSE (ft, NGVD)
L-3 Canal	Design	16.90
	SPF Flood	17.08
	SPF No Flood	16.10
Miami Canal	Design	11.50
	SPF Flood	10.50
	SPF No Flood	10.50
STA-5 Discharge	Design	12.15
	SPF Flood	10.78
	SPF No Flood	11.91
STA-6 Discharge	Design	14.85
	SPF Flood	16.64
	SPF No Flood	15.57
Southern	Design	12.30
	SPF Flood	12.80
	SPF No Flood	12.80

Table 5-2
Build Out Configuration 1-D Modeling Boundary Conditions

Model Name	Flow Condition	Downstream WSE (ft, NGVD)
L-3 Canal	Design	16.80
	SPF Flood	16.34
	SPF No Flood	16.34
Miami Canal	Design	11.50
	SPF Flood	10.50
	SPF No Flood	10.50
STA-5 Discharge	Design	12.15
	SPF Flood	10.78
	SPF No Flood	11.91
STA-6 Discharge	Design	12.00
	SPF Flood	12.00
	SPF No Flood	12.00
Southern	Design	12.30
	SPF Flood	12.80
	SPF No Flood	12.80

Head losses through the various hydraulic control structures were modeled using Culvert Master software developed by Haestad Methods. Preliminary structure sizes and numbers were estimated for the modeling effort based on the assumption that the maximum WSE rise in a treatment cell should be no more than about 1.5-foot above the Design Flow stage when subjected to the Standard Project Storm event. The 1.5-foot allowable WSE rise was based on preliminary wave runup results of about 1.4 feet for the cells and a 3-foot reservoir freeboard. The relatively high starting WSE produced by the Design Flow and the limitation of 1.5-foot WSE rise are conservative conditions that are unlikely to be exceeded.

The headloss through the existing outlet structures for STA-6 Flow-ways 3 and 5 were modeled using Culvert Master software and a spreadsheet analysis of the hydraulic interactions. The analyses resolved the interaction of weir, orifice, and culvert flow that govern the flow capability of these structures. Graphs in Appendix B show the interaction of the three flow components. Under the conditions imposed in the Initial Configuration, flow through the structures was controlled by culvert capacity. Under the Build Out Configuration, the reduced WSE in the STA-6 Discharge Canal makes weir flow the controlling factor for the structures.

The Flow Rating Tables and Structure Description Output for each of the various structures found in the Initial and Build Out Configurations are shown in Appendix B.

5.2.2 Two-Dimensional Modeling

Two-dimensional (2-D) modeling is required to accurately model the flow of stormwater through the relatively shallow constructed treatment cells. The project's Initial Configuration includes four constructed wetlands designated STA-5 Flow-way 3, STA-6 Section 2, STA-6 Section 1, Cell-3, and STA-6 Section 1, Cell 5. STA-5 Flow-way 3 is divided into two sub-cells to help control stages and improve the operational characteristics. The project Build-Out includes the additional constructed wetlands STA-5 Flow-way 4, STA-5 Flow-way 5, and STA-6 Section 4. STA-5 Flow-ways 4 and 5 are also subdivided by internal levees.

Two-dimensional depth-averaged hydrodynamic models were developed for the STA-5 Flow-ways 3, 4, and 5 and STA-6 Sections 2 and 4 cells to help in the design and evaluation of the individual cell's hydraulic performance. Existing 2-D models for STA-6 Section 1, Cell-3 and STA-6 Section 1, Cell 5 were modified by URS for this analysis (Sutron, 2004). The inlet and outlet configurations were modified to accommodate the proposed inlet modifications. Existing 2-D models for STA-5 Flow-ways 1 and 2 prepared by Burns and McDonnell were used to reassess the head losses through those systems under the proposed flows (Burns and McDonnell, 2004). The modeling analyses were conducted using the FESWMS finite-element model. The SMS 8.1 software system was used to develop model inputs and for post-processing of the simulation results.

The modeling was implemented for three primary purposes:

- determine the head loss across each cell for each set of design flow scenarios

- determine the average depth and wetted area in each cell
- evaluate flow patterns within each cell

The tailwater elevations for downstream treatment cells were calculated as follows. The WSE calculated by the HEC-RAS modeling in the downstream canal was added to the outlet structure headloss calculated for each flow condition. This provided the downstream treatment cell's tailwater elevations. The 2-D modeling then predicted the WSEs at the cell's upstream end for each flow condition. These elevations were then added to the inlet structure's headloss for each flow condition. This provided the tailwater elevations for the upstream treatment cell. Calculation of the remainder of the flow-way's water surface profile was performed, as above.

Individual models were developed for each cell in the STAs. For the proposed STA-5 Flow-way 3, models for cells 3A and 3B were created. STA-5 Flow-way 3A is upstream of STA-5 Flow-way 3B and flows from STA-5 Flow-way 3A to STA-5 Flow-way 3B are controlled by two gated culverts, which breach a levee that runs between the two cells. The models for STA-5 Flow-way 4 and 5 were divided into Sections A and B also. A model was also developed for the proposed STA-6 Section 2 and Section 4 areas. Calibrated FESWMS models for STA-5 Flow-ways 1 and 2, STA-6 Section 1, Cell-3 and STA-6 Section 1, Cell 5 were already created in previous studies (Sutron, 2004), (Burns and McDonnell, 2004). The digital model files were obtained and subsequently modified for use in these analyses. All other FESWMS models were developed specifically as part of this analysis.

5.2.2.1 Model Input Data

The application of the 2-D FESWMS model requires data for:

- topography
- frictional characteristics (Manning's "n")
- lateral diffusion characteristics

Topography

Two sources of data were available during the modeling analysis. The first data set was a topographic map of the area provided by USSC in digital form (USSC, 1989). No datum was provided with the data. It was assumed to be NGVD, but could not be verified by USSC due to the map's age. This map contained only the large-scale features of the relatively flat natural terrain.

The second data set, which was obtained specifically for this project, was survey data collected by Weidener & Associates for STA-5 Flow-way 3 and STA-6, Section 2. This data set included points in the relatively flat natural terrain, but focused mostly on levee and ditch elevations. Due to Compartment C still being in sugar production, accurate ground surveys for portions of the areas were not available for this study. The project-specific survey data was provided as an ASCII xyz point file in the NAVD datum.

Conversion to NGVD was made prior to use with the web-based National Geodetic Survey VertCon conversion tool (www.ngs.noaa.gov/TOOLS/Vertcon/vertcon.html).

The construction of the STAs will involve removing many of the existing internal and levee and ditch features and adding various internal levees, and feeder and collection ditches. Features running parallel to the flow will be removed while ditches and canals running perpendicular to the flow will act to distribute flow across the cell perpendicular to the flow. Berms running perpendicular to the flow direction are expected to be degraded to no higher than about 6 inches above the local grade. In the SAV cells, these perpendicular ditches may also provide some benefit to reducing wave setup and where higher topography is left in place, localized areas of emergent vegetation will provide beneficial/variable habitat areas. Therefore, the two topographic data sets were modified to remove features with elevations that deviated from the natural terrain. In constructing the models, the modified topographic data sets were used to set the topography associated with the natural terrain. Then the proposed feeder canals, collection ditches and borrow ditches were added to the models.

The elevations for the large-scale natural terrain in the two survey data sets compared well. For the purposes of using the topographic data to configure the individual cell models, we selected the USSC map data for the initial STA-6 Section 2, Section 4, and the STA-5 Flow-way 4 and 5 models, and we used the project-specific data for the STA-5 Flow-way 3 cell models. Although the project specific data is preferred, it contained only limited points in the natural flat terrain areas due to flooding and inaccessibility in STA-6, Section 2. Project specific survey data for the Flow-way 4 and 5 areas will not be available for this study due to on-going USSC farming operations.

Friction Parameters

Frictional effects are represented by a Manning's "n" formulation in the FESWMS model. We have adopted two values, one representing shallow water areas and one representing conditions in the feeder and collection ditches. In an analysis conducted for SFWMD (Burns & McDonnell, July 2004), a vertical profile of Manning's "n" was derived for conditions similar to those expected to occur in the shallow water areas of the proposed STA-5 Flow-way 3, 4 and 5 and STA-6 Section 2 and Section 4 cells. The friction profile was determined by calibrating the model to measured flow and stage data in STA-5-1 and STA-5-2. The adopted depth-dependent friction profile is shown in Figure 7, and consists of an 'n' value of 1.5 over the first 1.25 feet of water depth, which then decreases linearly to an 'n' value of 0.5 over the next 2.5 feet, and is constant at 0.5 deeper than 3.75 feet. In the ditches, the standard 'n' value for deep water flows of 0.025 was used (Chow, 1959). The STA-6 Section 1, Cell-3 and STA-6 Section 1, Cell 5 models include friction parameters that were established by calibrations conducted by Sutron (Sutron, 2004).

Diffusion

The FESWMS model represents lateral diffusion using a formulation comprised of a constant value plus a term that varies with the flow speed. There is no data available to calibrate the diffusion parameters, therefore we used values based on experience. A sensitivity analysis showed that the model results were not very sensitive over a reasonable range of diffusion parameter values, which is the expected result for relatively low velocity flows. For all simulations we used 5.0 ft^2/s for the constant coefficient and 0.6 (as suggested in the model literature) for the variable coefficient (FHWA, 2002).

5.2.2.2 Model Configurations

The boundaries for each of the individual cell models were obtained from the STA-5 construction drawings (Burns and McDonnell, STA-5 Contract Drawings, August 1997). All models were developed in Florida State Plane East coordinates in feet. After the model boundaries were specified, a finite-element mesh was generated and the topography mapped onto the mesh. The mesh was designed to provide adequate lateral resolution of the feeder and collection ditches and borrow pits that were part of the planned design. After the topography was mapped onto the mesh, the elevations in the areas of ditches and pits were adjusted according to the design plans.

Each cell model contained flow boundary conditions on the western boundary and WSE boundary conditions along the eastern boundary. The specific locations for the boundaries were set to coincide with the planned flow control structures.

The mesh for the STA-5 Flow-way 3A cell is shown in Figure 8. It is about 12,800 feet long in the east-west direction and 5,200 feet wide. The flow-way width is similar to the existing STA-5 Flow-ways 1 and 2.

A high-resolution mesh was developed for the feeder canals, the collection ditches, and the borrow pits along the north and south boundaries of the proposed STA-5 Flow-way 3. The feeder canal is approximately 40 ft wide, extends eastward from the west boundary about 3,000 feet, and then crosses the cell. It has a trapezoidal cross-section with 8 ft depth and 8 ft wide base. It was created on the mesh by subtracting 8 feet from the natural terrain elevation for mesh nodes coincident with the base of the canal. The collection ditch is about 30 feet wide, and is offset from the eastern boundary by 30 feet. It has a trapezoidal cross-section 30 feet wide, with and a base elevation of 1 foot NGVD. The borrow pits were about 40 feet wide, 500 to 600 feet long and staggered with 500 to 600 foot spacing. The borrow will be used to construct levees along the southern edge of the cell. The borrow pits have a triangular cross-section with bottom elevations set to 0 feet NGVD. Figure 9 shows the topography as it was interpolated onto the mesh, including the canal and pit features.

The mesh in the southwestern corner of the mesh was adjusted so that the edges of elements coincided with the topographic contours. This was done because it was expected that parts of the domain in this area may not be wetted under certain flow conditions, and the alignment of the mesh with the contours improved the convergence of the numerical solution techniques used by the FESWMS model.

Flow boundary conditions were set at the western end of the feeder canals and elevation boundary conditions were set at two outlet locations along the eastern boundary, such that each was centered on a one-half section of the boundary. The collection ditch bottom elevation was continued through the offset to the eastern boundary in the mesh cells containing the WSE boundary conditions.

Meshes for the STA-5 Flow-way 3B, 4A and 4B, 5A and 5B and STA-6 Section 2 and Section 4 Cells were constructed using the same techniques as for STA-5 Flow-way 3A. The mesh for STA-5 Flow-way 3B is shown in Figure 10 and the final topography in Figure 11. The cell is about 7,900 feet long in the east-west direction and about 5,200 feet wide. A feeder canal with a triangular cross-section, 30 feet wide with a bottom elevation of 5 feet runs along the western boundary. A collection ditch runs parallel to the eastern boundary with a triangular cross-section 30 feet wide and 4 ft deep. The collection ditch is offset 30 feet to the west of the eastern boundary. Two flow inlet boundaries were created, and placed to coincide with the position of the WSE elevation boundaries of the STA-5 Flow-way 3A model. Along the western boundary of the STA-5 Flow-way 3B model, two WSE outlet boundaries were specified, similar to the approach used for STA-5 Flow-way 3A. The topography was also adjusted slightly to represent the terrain associated with the farm plots within the STA-5 Flow-way 3B cell. At approximately quarter-mile intervals, 20-foot wide low-relief ridges were added to the mesh. A cross-section of the natural terrain at the location of each added ridge is shown in Figure 12. The ridges were set with elevations between 12 and 12.5 ft and are evident in the topographic contours in Figure 11. This ridge treatment was only implemented in the STA-5 Flow-way 3B model. Sensitivity testing indicated that the treatment did not significantly affect the headloss results at the Design and SPF flow rates and WSEs. The ridge treatment is expected to have more effect during lower flows and levels.

The remaining models for STA-5 Flow-ways 4A, 4B, 5A and 5B were constructed in the same way as for STA-5 Flow-way 3A and 3B (excepting ridges). Similar feeder canals and collection ditches were represented in each cell. The topography for STA-5 Flow-way 4 and 5 (both A and B cells) had borrow pits on both the north and south edges. The STA-5 Flow-way 5A and 5B cells were connected with three (rather than two) gated structures, and had two gates at the downstream end of STA-5 Flow-way 5B. Also, the feeder canal for STA-5 Flow-way 5A was generally aligned parallel to the 15-foot NGVD contour in the cell. The meshes and topography for each model is shown in Figures 13 through 20.

The mesh for STA-6 Section 2 is shown in Figure 21 and the cell topography in Figure 22. The cell is about 7,800 feet wide, in both directions. A feeder canal with a triangular cross-section, 30 feet wide with a bottom elevation of 5 feet runs along the western boundary. A collection ditch runs parallel to the eastern boundary with a triangular cross-section 30 feet wide and 4 ft deep. The collection ditch is offset 15 feet to the west of the eastern boundary. Three flow and WSE elevation boundaries (representing the three inlet and outlet structures) were specified along the western and eastern boundaries, respectively. These were placed such that each boundary would be centered on a one-third section of the boundary.

The STA-6 Flow-way 4 model mesh and topography are shown in Figures 23 and 24. The STA-6 Flow-way 4 cell has a similar collection ditch to that of the STA-5 cells, but the feeder ditch was adjacent to the western levee of the cell and only extended partly down the boundary. The inlets to the cell are at the northwestern corner, and on the western edge at the southern end of the feeder canal. The outlets from the cell are the inlet structures used for STA-6, Section 2.

The mesh and topography for the STA-6 Section 1, Cell-3 and STA-6 Section 1, Cell 5 cell models are shown in Figures 25 and 26. These meshes were obtained by modifying the existing models (Sutron, 2004). The modifications consisted of removing the inlet and outlet weir structures, and inserting separate inlet and outlet boundaries. The existing inlet canal was assumed to remain with new gated culverts feeding into this ditch. The inlet canal was assumed to be blocked between the cells so they could be operated independently. The levee between each cell and the inlet canal was assumed to be breached in order to allow free inflow. The inflow locations remain the same as the existing locations.

5.2.3 Flow Cases

Six steady-state flow cases were developed for analysis. These scenarios consist of tail-water elevations and flow conditions. The cases for each cell are summarized in the Tables 5.3 to 5.9 below.

Table 5.3 Boundary Conditions for STA-5 Flow-way 3 Flow Cases

Flow Case	Flow Rate (cfs)	STA-5-3A WSE (ft)	STA-5-3B WSE (ft)
Initial Configuration Design	596	15.4	14.1
Initial Configuration SPF w/o Miami Flood	836	16.0	15.3
Initial Configuration SPF w/Miami Flood	360	14.0	13.5
Build Out Configuration Design	381	14.8	13.5
Build Out Configuration SPF w/o Miami Flood	500	15.1	13.5
Build Out Configuration SPF w/Miami Flood	360	14.8	13.5

Table 5.4 Boundary Conditions for STA-6 Section 2 Flow Cases

Flow Case	Flow Rate (cfs)	WSE (ft)
Initial Configuration Design	667	15.1
Initial Configuration SPF w/o Miami Flood	1060	16.2
Initial Configuration SPF w/Miami Flood	700	16.9
Build Out Configuration Design	355	14.0
Build Out Configuration SPF w/o Miami Flood	605	14.0
Build Out Configuration SPF w/Miami Flood	762	14.0

Table 5.5 Boundary Conditions for STA-6 Section 1, Cell-3 Flow Cases

Flow Case	Flow Rate (cfs)	WSE (ft)
Initial Configuration Design	105	15.0
Initial Configuration SPF w/o Miami Flood	168	15.7
Initial Configuration SPF w/Miami Flood	111	16.8
Build Out Configuration Design	46	14.1
Build Out Configuration SPF w/o Miami Flood	78	14.2
Build Out Configuration SPF w/Miami Flood	92	14.3

Table 5.6 Boundary Conditions for STA-6 Section 1, Cell 5 Flow Cases

Flow Case	Flow Rate (cfs)	WSE (ft)
Initial Configuration Design	286	15.1
Initial Configuration SPF w/o Miami Flood	454	16.2
Initial Configuration SPF w/Miami Flood	301	16.9
Build Out Configuration Design	124	14.4
Build Out Configuration SPF w/o Miami Flood	209	14.6
Build Out Configuration SPF w/Miami Flood	246	14.7

Table 5.7 Boundary Conditions for STA-6 Section 4 Flow Cases

Flow Case	Flow Rate (cfs)	WSE (ft)
Build Out Configuration Design	355	15.1
Build Out Configuration SPF w/o Miami Flood	605	15.1
Build Out Configuration SPF w/Miami Flood	712	16.1

Table 5.8 Boundary Conditions for STA-5 Flow-way 4 Flow Cases

Flow Case	Flow Rate (cfs)	STA-5-4A WSE (ft)	STA-5-4B WSE (ft)
Build Out Configuration Design	390	15.9	14.0
Build Out Configuration SPF w/o Miami Flood	665	16.6	14.0
Build Out Configuration SPF w/Miami Flood	781	16.7	14.0

Table 5.9 Boundary Conditions for STA-5 Flow-way 5 Flow Cases

Flow Case	Flow Rate (cfs)	STA-5-5A WSE (ft)	STA-5-5B WSE (ft)
Build Out Configuration Design	490	15.7	14.5
Build Out Configuration SPF w/o Miami Flood	835	16.4	14.5
Build Out Configuration SPF w/Miami Flood	981	16.6	14.5

In each scenario, the flows were divided equally among the multiple flow inlets. The WSE boundary conditions were determined from conditions in the downstream canals obtained using the HEC-RAS modeling analysis for the surrounding canal system (see section 5.2 of this report). The WSE boundary conditions for upstream flow-ways were developed after the analysis of downstream flow-way and were calculated by adding the culvert head loss to the WSE calculated by FESWMS at the downstream flow-way inlets. The head losses for the culverts were calculated using Culvert Master software.

The District has indicted that the minimum desirable water depth in the SAV cells is 45 cm. In some flow cases for the 2-D modeling, the downstream WSE was elevated beyond that required by the downstream and outlet structure hydraulics to provide the minimum water depth at the cell's downstream end.

5.3 Water Quality Modeling

The phosphorus removal efficiency of the STA's is an important factor in determining the configuration and operation of the cells. Water quality modeling for the STA's was performed using the April 2002 version of the District's DMSTA modeling software (Walker and Kadlec, 2002). This software includes cell parameter datasets that quantify various factors for the phosphorus removal efficiency of wetland treatment cells.

The District is evaluating the minimum water depths that are desirable in the Emergent and SAV cells. For SAV cells, the District has indicated that 45 cm is the desirable minimum water depth. Sensitivity testing for the minimum water depth in the Emergent cells was performed for this study. Using the DMSTA software, analysis trials were conducted in which only the outflow control depth differed. For each flow-way, the predicted phosphorus outflow concentrations (geometric mean) were compared to determine the effect of the water depth in the Emergent cells. The results for the five STA-5 flow-ways are shown in Table 5.10. For all comparisons, the predicted phosphorus concentrations increased when the water depth was increased from 25 cm to 45 cm. The percentage increase ranged from 5.3 to 7.7 percent. The outflow concentrations from the SAV cells (flow-way discharge point) were used in the comparisons.

Table 5.10 Effect of Increased Water Depth in Emergent Cells

Flow-way	% Increase in Phosphorus Concentration
STA-5 Flow-way 1	5.9
STA-5 Flow-way 2	5.9
STA-5 Flow-way 3	7.7
STA-5 Flow-way 4	6.7
STA-5 Flow-way 5	5.3

Under the Initial Configuration of the proposed STA's, some flow from the C-139 Basin will be bypassed to the STA-6 system. For this analysis, the bypassed was assumed to occur when the C-139 Basin flows exceed 1080 cfs. This represents maximum flow capacity of 360 cfs each to STA-5 flow-ways 1, 2, and 3. The bypassed flow was added to the C-139 Annex Basin dataset. The phosphorus concentration of the combined flow was adjusted on a flow-weighted basis.

Where possible, cell-specific phosphorus removal parameters were used in the models (STA-6 Flow-ways 3 and 5). In other cases, generic phosphorus removal parameters were used. The generic parameter sets were developed from performance observations of many STAs. For the STA-5 cells, the generic phosphorus removal datasets EMERG and NEWS_2 were used to model the emergent and submerged aquatic vegetation (SAV), respectively. To aid in bounding the removal efficiency, additional modeling runs were conducted by substituting another generic phosphorus removal parameter set (SAV_C4) for the NEWS_2 parameters. Either of these SAV parameter sets should be applicable to the proposed SAV cells.

For the existing STA-6 flow-ways, local phosphorus removal parameters have been developed. These parameter sets were developed for the predominately emergent vegetated cells. The STA-6_5 parameter set was used for the emergent portions of STA-6 Section 4 and Section 1 Cell 5. The STA-6_3 parameter set was used for STA-6 Section 1, Cell 3.

The DMSTA software enables modeling treatment cells in parallel and series arrangements. Figures 3 and 4 show that most of the flow-ways are composed of cells in series. All the flow-ways parallel each other.

The model includes the surface area and width of the treatment cells. The variation in the cell's water surface elevation with respect to the influent rate is incorporated into the model via a minimum cell water depth input and flow coefficients. Since the cells are proposed or will be modified, there are no observed water surface elevations available to enable calibration of the flow coefficients. Typical coefficients cited by the software authors were used.

The model includes the ability to estimate the amount of water lost to seepage from the cells as well as the ability to estimate the amount of seepage recycled back into the cells. For the STA-5 Flow-ways 1, 2, and 3, seepage loss and recycling was included in the modeling. These flow-ways include seepage collection ditches. The remaining STA-5 and 6 flow-ways do not include or will not include seepage collection ditches. Therefore, these cells did not include any seepage losses or seepage recycling.

Additional modeling was conducted on certain sub-divided cells. The Long Term Plan proposed to divide the STA-6 Section 2 and Section 1 Cell 5 into emergent and SAV sub-cells. In these cells, the cell topography must be conducive to producing emergent and SAV vegetation types. In the case of the STA-6 Section 2 cell, it appears the topography may not slope enough to promote separate vegetation types in these cells. For STA-6 Section 1 Cell 5, there was enough topographic relief to promote the vegetation difference. For STA-6 Section 1 Cell 5, the STA-6_5 parameters were used for the emergent cell and both the NEWS_2 and SAV_C4 parameters were used for the SAV cell.

The modeling was conducted to assess the two distinct phases of the STA development. The Initial configuration includes the existing STA's along with the addition of STA-5 Flow-way 3 and STA-6 Section 2. The Build Out Configuration includes the remaining STA cells, STA-5 Flow-ways 4 and 5, and STA-6 Section 4. In the case of the STA-5 flow-ways, all are sub-divided into emergent and SAV cells.

Since the DMSTA software can accommodate a maximum of six cells, the Build Out configuration of the STA-5 system was modeled in two operations. Cells 1A, 1B, 2A, 2B, 3A, and 3B were modeled together in a DMSTA run. Cells 4A, 4B, 5A, and 5B were then modeled together. In both cases, the flow percentage directed into each cell train was in accordance with its area-weighted basis. The results from the two models were combined to assess the overall removal for the STA-5 system.

5.3.1 Modeling Input Variables

The surface area for the treatment cells was estimated from either maps produced by the District or from areas given in the Long Term Plan. The mean flow path width was estimated from the District's maps. Since good plug flow through the cells may not always occur, the *tanks-in-series* input for the model was set at 3 for all modeling. The tanks-in-series value reportedly ranges from 2 to 6 to represent poor to excellent plug flow through a cell, respectively. Three tanks-in-series is a typical value used in previous STA modeling.

The District's experience with STA operations indicates that there are desirable water depths for the various cell vegetation types. In the case of emergent vegetation cells, the outflow control depth was set at 25 cm. For SAV cells, the outflow control depth was set at 45 cm.

For the STA-5 Flow-ways 1, 2, and 3, seepage recycling was assumed to capture and return 50 percent of the seepage volume. Seepage parameters used to estimate the losses from the cells were based on typical seepage rates and seepage control depths used in previous modeling for STA-5.

The reservoir water residence time, maximum and mean inflow, maximum reservoir storage, reservoir phosphorus decay rate, bypass depth, and maximum outflow were unconstrained, allowing the model to set the values. Zero values in these input fields indicate the values are unconstrained. The maximum inflow to each cell was taken as the Flood SPF values shown in Figures 5 and 6. The flowrate shown in the figures in cfs was converted to hm3/day for input in the model.

The rainfall phosphorus concentration used in all modeling was assumed to be 10 parts per billion. The atmospheric phosphorus loading used in all modeling was assumed to be 20 milligrams per square meter per year. These values were taken from the modeling performed for the Long-Term Plan (SFWMD, 2004).

The initial water column concentration, initial phosphorus storage, and initial water column depth values were typical values used in previous modeling for the STA's. Since the modeling period covers almost 40 years, the initial values have very little effect on the results.

6.0 MODELING RESULTS

The modeling results describe the movement of water through and around the project. This report provides the results for all flow-ways in the STA-5/6 system. The modeling results were used to develop water surface profiles and control structure sizing for each cell.

6.1 Hydrologic Modeling Results

The peak flow and WSE rise in the proposed cells were evaluated using HEC-HMS. Using the SPS event, the response of the cells and adequacy of the outflow control structures was assessed. The analysis results shown in Appendix C show that the peak flow from the STA-6 Section 2 Cell is 2,200 cfs with a WSE rise of 1.2 feet over the design flow elevation. For STA-5 Flow-way 3A, the results show the peak outflow from the cell is 1,600 cfs with a WSE rise of 1.2 feet. For STA-5 Flow-way 3B, the results show the peak outflow from the cell is 2,000 cfs with a WSE rise of 1.3 feet. The assumed number and sizes of outflow structures are shown on the Appendix figures.

The hydrologic modeling results provide guidance for outflow structure sizing. Although the outflow structures can be sized using the design and SPF flows, this analysis method provides direct assessment of the required peak outflow capability and resulting WSE rise.

The number of structures was based primarily on maximum spacing of structures and desired redundancy. For cells with a width of about 1 mile, two structures were used. For 1.5 mile wide cells, three structures were used.

The number and size of structures developed as described above were used to develop the structure flow rating tables shown in Appendix B. The rating table information was then used to develop the water surface profiles for the cells.

6.2 One-Dimensional Modeling Results

HEC-RAS models for all canals were run using the calculated canal flows for each of the flow cases. Calibrated canal models were used when available, otherwise typical Manning's "n" values were used in the models to estimate the WSE in the canals. The results of the HEC-RAS modeling for the various canals and flow conditions are provided in Appendix D.

The WSE at the southern limit of the project area has significant effect on the WSE's in the STA-6 Discharge Canal, the L-3 Canal, and the L-4 Canal. The modeling assumed the G-88 and G-155 structures have been demolished. The calibrated Southern model was used to calculate the WSE at the project's southern limit under the various flow conditions. The calculated WSE's at the confluence of the STA-6 Discharge, L-3, L-4, and L-3 Extension Canals (also known as Confusion Corner) is provided in Table 6.1.

All WSE results provided in this report have been rounded to the nearest tenth of a foot. The HEC-RAS outputs for the Southern model are shown in Appendix D.

The STA-6 Discharge Canal is expected to operate with a gravity discharge to WCA-3A in the Initial Configuration. The modeling indicated that the G607 structure has insufficient flow capacity to allow it to remain in place across the STA-6 Discharge Canal. The G607 structure is composed of three 66" diameter and two 84" diameter culverts through an earthen embankment through the canal. The existing culverts will require replacement with a bridge or high-capacity culvert system to carry the proposed flows from either the Initial or Build Out Configurations. The STA-6 Discharge Canal modeling assumed that the G607 structure has been modified or removed to reduce head losses.

The STA-6 Discharge Canal modeling uses the WSE shown in Table 6.1 under the Initial Configurations as the downstream WSE for calculating the water surface profile up the canal. In the Build Out Configuration, the STA-6 Discharge Canal is expected to incorporate a discharge pump station to reduce the stage in the STA-6 Discharge Canal and pump into the L-4/L-3 Extension canals. An assumed headwater elevation of 12 feet NGVD in the discharge pump station pumping pool was developed by trial and error in order to enable the desired flow through the treatment cells without adversely elevating the WSE in the L-3 Canal. The pumping pool elevation of 12 feet NGVD was used as the downstream WSE for calculating the water surface profile up the STA-6 Discharge Canal.

Table 6.1 Calculated Water Surface Elevations at Confusion Corner

Flow Case	WSE (ft, NGVD)
Initial Configuration Design	14.9
Initial Configuration SPF w/o Miami Flood	15.6
Initial Configuration SPF w/Miami Flood	16.6
Build Out Configuration Design	15.2
Build Out Configuration SPF w/o Miami Flood	16.1
Build Out Configuration SPF w/Miami Flood	16.4

To assess the maximum WSE in the L-3 Canal at the confluence with the Deer Fence Canal, the calibrated L-3 Canal model was evaluated with the existing G-406 diversion structure in place in the canal and using the Flood SPF By-Pass flow of 2,360 cfs. Under that condition, the WSE at the confluence was calculated as 21.84 feet NGVD. This was assumed to be the worst-case WSE under the existing condition and was used as the limiting WSE at this location for subsequent modeling. The HEC-RAS output for this analysis is provided in Appendix D.

When the G-407A diversion structure was placed in the calibrated L-3 Canal model, the WSE was found to exceed 21.84' in the Initial Configuration with SPF flow and the

Miami Canal was in Flood Condition. The additional backwater WSE created by the proposed G-407A structure significantly reduced the flow through the G-406 structure resulting in a further elevated stage upstream of G-406. In an attempt to reduce the WSE at Deer Fence Canal, the G-407A structure size was increased to a triple-barrel, 10' by 10' box culvert configuration. The WSE still exceeded the existing condition at the Deer Fence Canal confluence. In order to drop the WSE to allowable limits, the crest elevation of the G-406 diversion structure was reduced from 21.75' NGVD to 20.5' NGVD. This allowed the G-407A structure to be constructed identically to the G-406 structure (twin, 10' by 9' box culverts). The minor reduction in the G-406 crest height is expected to be less costly than constructing a significantly larger G-407A structure. The earthen dike can be excavated using a bulldozer or grader to reduce the crest height. The revised G-406 configuration was used for the Build Out Configuration of the L-3 Canal.

The WSE in the Miami Canal is maintained by the operation of the S-8 pump station. The operation rules for that facility provide guidance for the expected headwater elevation in the pumping pool. The District's Operation and Maintenance Division has updated the operation rules for the pump station. The District's operation rules now state that the pumps will be operated when the headwater WSE exceeds 11.5 feet NGVD during dry conditions and 10.5 feet NGVD during wet conditions (SFWMD, 2004). A downstream WSE of 11.5 feet was used in the model for the Design Flow condition in the modeling. A downstream WSE of 10.5 feet was used in the model for the SPF Flow conditions. The modeling assumes the only flow in the Miami Canal is the flow from the STA-5 Discharge Canal. All other Miami Canal flow from the north is diverted to the STA3/4 system. The HEC-RAS output for this analysis is provided in Appendix D.

The calculated WSE at the upstream end of the Miami Canal model was used as the downstream WSE in the STA-5 Discharge Canal model. The STA-5 Discharge Canal model provided WSEs at all STA-5 cell outlet locations. Discharge from the STA-5 Discharge Canal to the Rotenberger Tract via the G410 pump station was not included in this modeling. This pump station is primarily intended to supplement water supply to the tract during periods of drier periods and would not be operated during the project flood conditions. This assumption will produce a more conservative (higher) WSE in the STA-5 Discharge Canal. The calculated WSEs are shown in Table 6.2 in the STA-5 Flow-way 3B Downstream Water Surface Elevation listing. The HEC-RAS output for this analysis is provided in Appendix D.

The L-3 Canal model was used to calculate the WSEs at various points along the canal corresponding to cell inlet locations. The L-3 Canal model used for the Initial Configuration analyses included both the G-406 and G-407A diversion structures. For flow conditions when by-pass flow through the G-407A structure will occur, the WSE calculated in the Southern model (Table 6.1) was used as the downstream WSE in the L-3 Canal model. When by-pass is not expected, the downstream WSE was selected to produce a WSE of about 17 feet NGVD at Deer Fence Canal. The WSE of 17 feet is the District's desired maximum at that location. The HEC-RAS output for this analysis is provided in Appendix D.

Table 6.2
Water Surface Profiles

STA5 Flow-way 1A

Design Scenario	Initial Configuration Design	Initial SPF w/o Miami Flood	Initial SPF w/ Miami Flood	Buildout Configuration Design	Buildout SPF w/o Miami Flood	Buildout SPF w/ Miami Flood
Cell Flow (cfs)	596	836	360	381	500	360
Canal Water Surface Elevation (ft, NGVD)	17.2	18.2	16.2	16.2	16.8	16.2
Cell Inlet-Side Water Surface Elevation (ft, NGVD)	16.7	17.2	16.0	16.0	16.4	16.0
Cell Outlet-Side Water Surface Elevation (ft, NGVD)	14.9	16.4	13.8	13.9	14.1	13.8
Downstream Water Surface Elevation (ft, NGVD)	14.8	16.2	13.7	13.8	14.0	13.7

Table 6.2
Water Surface Profiles

STA5 Flow-way 1B

Design Scenario	Initial Configuration Design	Initial SPF w/o Miami Flood	Initial SPF w/ Miami Flood	Buildout Configuration Design	Buildout SPF w/o Miami Flood	Buildout SPF w/ Miami Flood
Cell Flow (cfs)	596	836	360	381	500	360
Upstream Water Surface Elevation (ft, NGVD)	14.9	16.4	13.8	13.9	14.1	13.8
Cell Inlet-Side Water Surface Elevation (ft, NGVD)	14.8	16.2	13.7	13.8	14.0	13.7
Cell Outlet-Side Water Surface Elevation (ft, NGVD)	14.5	15.9	12.5	13.1	13.3	12.5
Canal Water Surface Elevation (ft, NGVD)	14.3	15.5	12.0	13.0	13.2	12.0

Table 6.2
Water Surface Profiles

STA5 Flow-way 2A

Design Scenario	Initial Configuration Design	Initial SPF w/o Miami Flood	Initial SPF w/ Miami Flood	Buildout Configuration Design	Buildout SPF w/o Miami Flood	Buildout SPF w/ Miami Flood
Cell Flow (cfs)	596	836	360	381	500	360
Canal Water Surface Elevation (ft, NGVD)	17.3	18.7	16.1	16.2	16.8	16.2
Cell Inlet-Side Water Surface Elevation (ft, NGVD)	16.8	17.7	15.9	16.0	16.4	16.0
Cell Outlet-Side Water Surface Elevation (ft, NGVD)	16.2	17.3	15.1	15.2	15.7	15.1
Downstream Water Surface Elevation (ft, NGVD)	16.1	17.1	15.0	15.1	15.6	15.0

Table 6.2
Water Surface Profiles

STA5 Flow-way 2B

Design Scenario	Initial Configuration Design	Initial SPF w/o Miami Flood	Initial SPF w/ Miami Flood	Buildout Configuration Design	Buildout SPF w/o Miami Flood	Buildout SPF w/ Miami Flood
Cell Flow (cfs)	596.0	836.0	360.0	381.0	500.0	360.0
Upstream Water Surface Elevation (ft, NGVD)	16.2	17.3	15.1	15.2	15.7	15.1
Cell Inlet-Side Water Surface Elevation (ft, NGVD)	16.1	17.1	15.0	15.1	15.6	15.0
Cell Outlet-Side Water Surface Elevation (ft, NGVD)	14.1	15.3	12.5	12.7	12.8	12.5
Canal Water Surface Elevation (ft, NGVD)	13.9	14.9	11.8	12.6	12.7	11.8

Table 6.2
Water Surface Profiles

STA5 Flow-way 3A

Design Scenario	Initial Configuration Design	Initial SPF w/o Miami Flood	Initial SPF w/ Miami Flood	Buildout Configuration Design	Buildout SPF w/o Miami Flood	Buildout SPF w/ Miami Flood
Cell Flow (cfs)	596	836	360	381	500	360
Required Inlet Canal Water Surface Elevation (ft, NGVD)	17.5	18.6	17.1	16.7	17.2	16.6
Cell Inlet-Side Water Surface Elevation (ft, NGVD)	17.0	17.6	16.9	16.5	16.8	16.4
Cell Outlet-Side Water Surface Elevation (ft, NGVD)	15.5	16.5	14.8	14.8	15.1	14.8
Downstream Water Surface Elevation (ft, NGVD)	15.3	16.3	14.8	14.8	15.1	14.8

Table 6.2
Water Surface Profiles

STA5 Flow-way 3B

Design Scenario	Initial Configuration Design	Initial SPF w/o Miami Flood	Initial SPF w/ Miami Flood	Buildout Configuration Design	Buildout SPF w/o Miami Flood	Buildout SPF w/ Miami Flood
Cell Flow (cfs)	596	836	360	381	500	360
Upstream Water Surface Elevation (ft, NGVD)	15.5	16.5	14.8	14.8	15.1	14.8
Cell Inlet-Side Water Surface Elevation (ft, NGVD)	15.3	16.3	14.8	14.8	15.1	14.8
Cell Outlet-Side Water Surface Elevation (ft, NGVD)	14.7	16.1	13.5	13.5	13.7	13.5
Outlet Canal Water Surface Elevation (ft, NGVD)	14.5	15.8	12.2	13.1	13.4	12.1